Seismic Response of Highway Embankments

J. DAVID ROGERS

The basic mechanisms by which natural slopes and highway embankments respond to seismic loading are explained. The response of a slope to earthquake loads depends on a number of localized conditions, such as geologic structure, topographic setting, seismologic setting, complexity of saturated zones, and the geophysical properties of soil and rock making up the mass. Observations following earthquakes suggest that slope response varies widely depending on the above-cited factors. In conclusion, how to draw the distinction between landslide and settlement-induced movement is discussed.

Through the first few generations of highway construction (1925–1955), there was scant engineering geologic input as to the routing of roads. As a consequence, much of the U.S. highway infrastructure is founded upon or adjacent to geologically unstable ground, terrain that may be particularly susceptible to reactivation through seismic loading. Figure 1 shows a schematic representation of a typical ancient landslide complex in mountainous terrain. Such complexes often lie within regions of recognized paleoseismic activity, and the potential for seismic instability is considered high. However, the accurate prediction of seismically induced reactivation of such deposits requires a thorough understanding of the various mechanisms of mass movement.

MECHANISMS OF SEISMICALLY INDUCED MOVEMENTS

Seismically induced slope movements have often been categorized as lurching, lateral spreading, liquefaction, and enhanced ground shaking. "Lurching" is a colloquial term used to describe permanent ground movements resulting from earthquakes. No distinction is made as to the mechanism of movement, which could be due to partial liquefaction, differential settlement (densification), or landslide reactivation. "Lateral spreading" is the more correct term used to describe gross lateral distortions on sloping ground, generally referring to natural as well as man-made embankments. The slope of the ground may be very slight, as little as 0.50 degree. Liquefaction is a failure mode in which pore water pressures are developed that exceed the effective strength of a porous material through buoyance (the effective confining pressure is lessened by the amount of increased pore pressure). In many instances, liquefaction may be confined to a particular stratum and thereby engender lateral spreading or lurching of higher ground.

"Enhanced ground shaking" is also a colloquial term, used to describe local enhancement of incoming seismic energy due to wave deformations induced by near-surface deposits of low modulus soil or soils. Generally, these distortions are greater in the higher vibrational frequencies (above 5 Hz), and lower in the frequencies associated with large quakes (closer to 1 Hz). Wood (*1*) originally described ground enhancement effects in reporting increased shaking intensities along the margins of San Francisco Bay during the 1906 earthquake. Naito (*2*) later made almost identical observations during the disastrous 1923 Tokyo quake. Much later, Borcherdt (*3*) did extensive experimental work in studying this phenomenon in the San Francisco Bay area, and Seed and Idriss (*4,5*) subsequently incorporated such precepts into modern geotechnical earthquake engineering theory and practice. Today, nationwide standards for performing deterministic assessments are readily available throughout the United States (*6*). Following are brief discussions of those mechanisms most pertinent to landsliding.

Dynamically Induced Settlement

When large dynamic loads, such as earthquakes, transmit through a fill embankment, some densification invariably occurs because of the large shear stresses that are suddenly imposed on the embankment. Makdisi and Seed (*7*) were among the pioneers in providing simplified finite-element procedures for estimating embankment deformations induced by strong shaking motion. Less sophisticated pseudostatic assessments, such as those originally proposed by Seed and Martin (*8*), have generally been found lacking in predicting embankment response.

The effect of seismic wave excitation on embankments depends to a great degree on the following factors:

1. Geologic site conditions and whether subsurface embankment conditions (such as being situated along a bay margin or across an old channel or lake deposit) are favorable for producing site enhancement effects;
2. Number of equivalent load cycles in excess of about 0.10 g;
3. Ratio of horizontal to vertical acceleration;
4. Cohesive character of embankment materials (plastic embankments tend to damp and absorb more energy, whereas granular fills are more prone to densification);
5. Position of the water table within the embankment at the time of shaking (some larger embankments may have several perched levels);
6. Degree of available subdrainage built into the embankment (which helps alleviate excess pore pressures);

Rogers/Pacific, Inc., 396 Civic Drive, Pleasant Hill, Calif. 94523-1921.

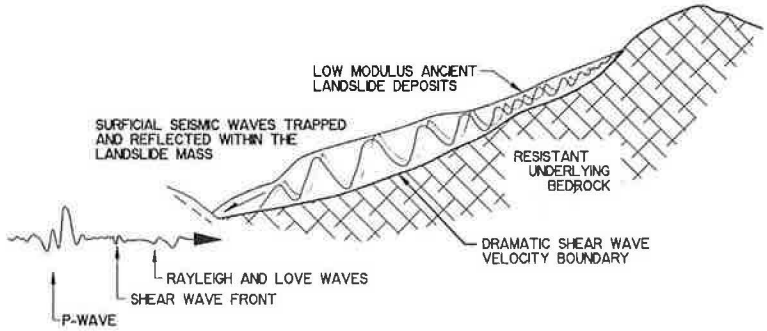

FIGURE 1 Site response of large ancient landslides in mountainous terrain, showing entrapment of surface waves in softer, yielding sediments or landslide cover.

7. Amount of previous densification induced by antecedent shaking, age of embankment, or previous cycles of groundwater rise and fall;
8. Seismologic constraints, such as near-field effects, wave entrapment, shallow bedrock escarpment reflections, Moho reflections, seismic focusing, bilateral versus uniaxial rupture scenarios at the hypocenter, and so on; and
9. Coseismic vibrations induced by adjacent ground movement, tidal waves, seiches, and so on.

Experience has shown that highway embankments are particularly susceptible to earthquake-induced settlements, especially bridge abutment approach fills (Figure 2). In the abutment area, large strains can be induced by vertical seismic acceleration and by transverse response of stiff bridge support members (the shorter supporting columns or bents will generally be of much higher stiffness).

Also particularly susceptible to quake-induced densification are older side-cast roadway fills across former gullies and ravines in steep, mountainous terrain (Figure 3). Valley fills are subject to settlement in directions transverse and parallel to the old valley trend (Figure 4). In such cases, the magnitude of observed settlement depends markedly upon strong motion duration and the water table position within the embankment. Embankments that are excited by seismic shaking in dry years tend to be most prone to dynamically induced settlement. On the other hand, earthquake loading effects on embankments are most pronounced at levels in excess of 80 percent of the static rupture strength, meaning that a large number of lower-level load cycles may have little or no effect on inducing large permanent strains (*7*).

Seismic Activation of Landslide Complexes

Most ancient slides exhibit abundant evidence of semicontinuous, long-term movement, or creep. As the slide mass is excited by either vertical or lateral earthquake accelerations, some densification inevitably occurs within units of low or moderate relative density. In addition, seismic shear waves induce excessive shear stresses, which cause the embankment to physically deform. Some portion of this physical deformation is not recoverable and results in permanent deformation, because soil and rock mixtures are elastoplastic. Dynamically induced settlements observed after earthquakes are, therefore, generally caused by a combination of mass densification and excessive shear stress. Embankments by their nature are constructed in a layered fashion, thereby engendering anisotropy of their physical properties and moduli of deformation (*9*). In addition, embankments are seldom symmetrical with respect to their foundations and as a consequence cannot be expected to show a symmetrical response. For these reasons, differential settlement of embankments or ancient landslide deposits is inevitable.

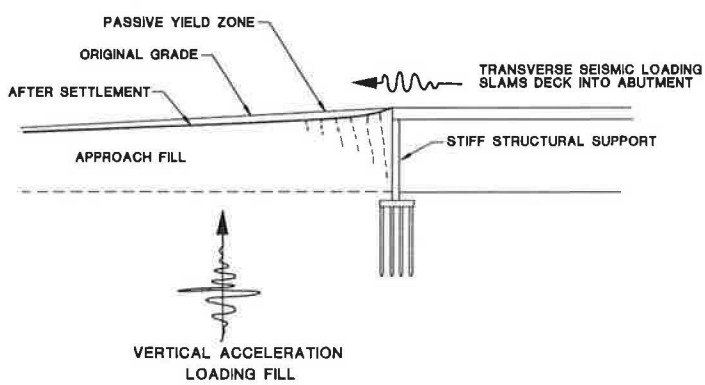

FIGURE 2 Dynamically induced settlement of viaduct approach fills.

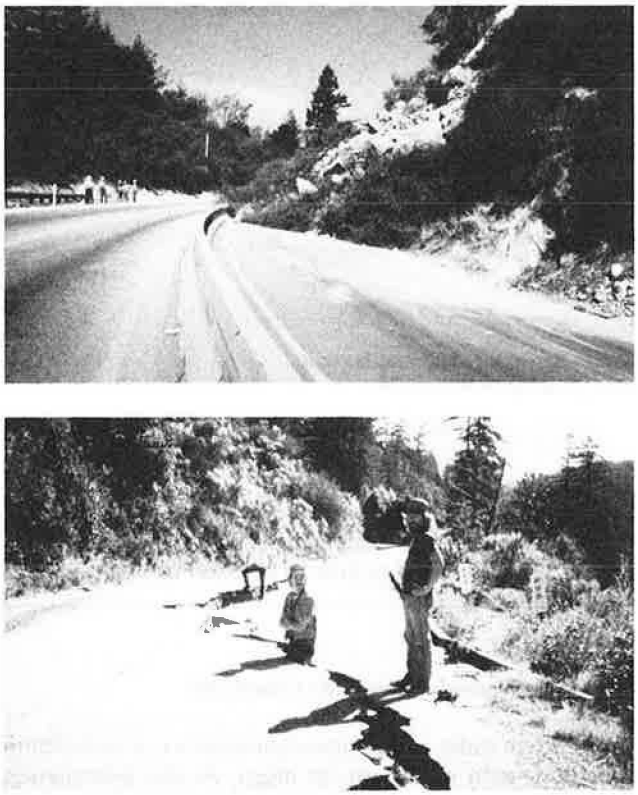

FIGURE 3 Roadway fills susceptible to quake-induced densification. *Top*: seismically induced rockfall of dry bedrock cut on California Route 17 near the epicenter of 1989 Loma Prieta earthquake (*photograph by Woodrow Higdon, Geo-Tech Imagery Int.*). *Bottom*: settlement of unkeyed fill wedge on Old Santa Cruz highway after 1989 Loma Prieta earthquake (*photograph by Dan Orange, University of California, Santa Cruz*).

Within the accepted precepts of soil mechanics consolidation theory, even dynamically induced settlement must follow lines of principal effective stress, which are not vertically inclined (although these lines are influenced by the ratio between horizontal to vertical acceleration). The lower the relative water table within a hillside compartment, the greater the potential for densification and classic settlement (because of higher effective stresses under dry conditions).

Differential densification will usually occur with a significant horizontal component of motion, causing tensile separations to form at the location of discrete but preexisting tension scarps. Often, these separations are most dramatic in old tension grabens, seen as soil-filled scarps or grabens on closer subsurface examination (Figure 5). The soil-filling of these scarps suggests that the same sort of tensile separation, settlement, and apparent downslope translation have occurred many times in the past. Downhole instruments, such as inclinometers, could be expected to register pseudo downslope motion in the event of simple densification because of the significant downslope component of movement (which would increase in accordance with slope inclination).

Graphic evidence of both consolidation and creep-induced settlement of large landslide complexes can be drawn from successive inclinometer readings. In Figure 6 a series of inclinometer readings and observations drawn over a period of 7 consecutive years is shown schematically. The uppermost cross section depicts observed groundwater levels shortly after the apparent cessation of macroslide movement. At this juncture, inclinometers were installed to record any further movement. The middle section shows observed water levels within the slide mass and indicated motion on one of the inclinometers 3 years later. Some net downslope motion appears to have occurred concurrent with an observed drop of the entrapped groundwater level. The lowest section shows the same situation some 7 years later. The inclinometer reading indicates

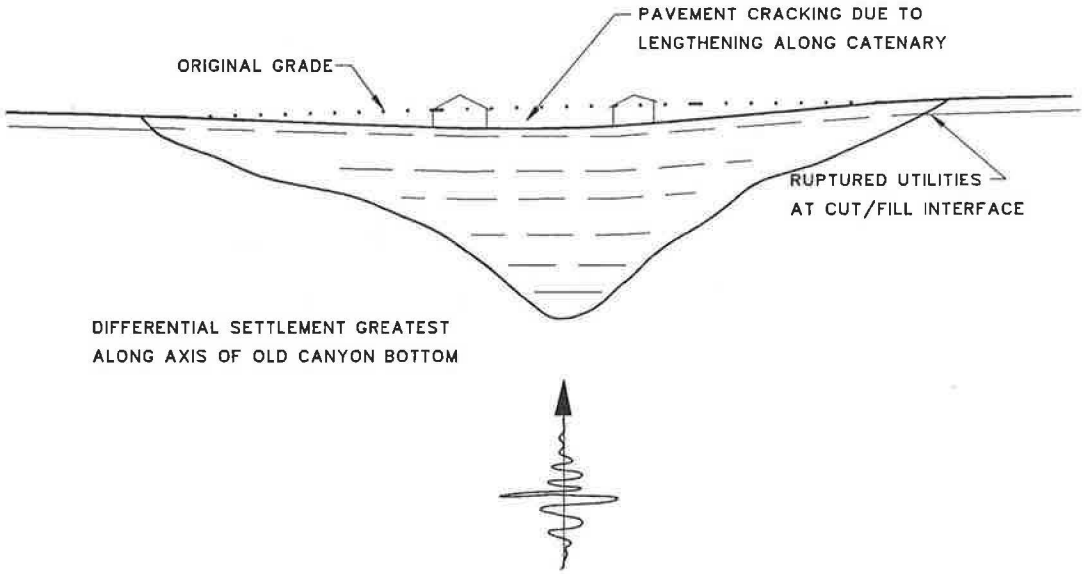

FIGURE 4 Dynamically induced settlement of valley fill.

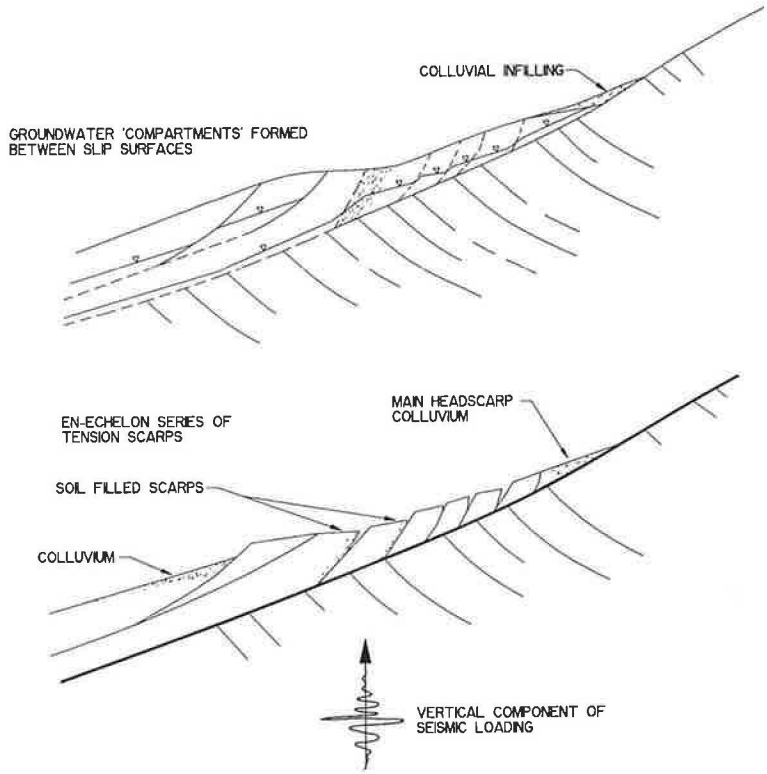

FIGURE 5 Differential densification of headscarp in ancient landslides. *Top*: coalescing ancient landslide. *Bottom*: pseudo slippage observed following earthquake.

more than twice the movement of 4 years before, and the water table has consistently continued to fall.

The curvilinear portion of the inclinometer profile appears to follow the declining water table. It is believed that the inclinometer has recorded consolidation and shrink-induced settlement of the slide mass, following dilation and swell associated with mass translation. Although significant subsurface motion is also recorded, it does not appear attributable to translation of the landslide mass on its own slip surfaces. Nevertheless, such movement, or macro creep, could be expected to shift any structure extending across the mass onto more stable adjacent ground. This would therefore be an example of pseudo landslide motion, induced by a combination of shrink, creep, and settlement as the slide mass slowly dries out and consolidates.

In general terms, the response of large ancient slides to seismic loading would appear to be heavily influenced by the following factors:

1. The relative static safety factor (SF) at the time of seismic shaking; no slide mass with a static SF of 1.70 or greater has failed in an earthquake, no matter of what intensity (*10,11*);
2. The relative amount of groundwater entrapped within the slide mass at the time of shaking; earthquakes during years of high precipitation tend to reactivate many more slides than those occurring in dry years;
3. The geographic position of the slide mass with respect to the earthquake focal area; and
4. The geophysical properties of the landslide mass and the duration of strong shaking (number of equivalent load cycles).

The latter two factors were discussed previously. The relative position of entrapped groundwater tables is critical to static slope stability assessments and probably even more critical to seismic stability evaluations. In general, higher levels of moisture, or saturation, tend to occur in zones subjected to downslope accretion (as shown in Figure 1) or adjacent to structural aquacludes, such as faults.

A schematic cross section of a hypothetical headscarp is shown in the upper part of Figure 5. Individual groundwater compartments are typically formed between both lithologic and old slip surface boundaries. Old tension scarp grabens, typically exposed as areas of gentle slope or closed depressions, usually funnel large amounts of surface run-off into the slide mass.

RESPONSE OF EMBANKMENTS

In order to study the deformational potential of highway embankments, some basic understanding of static effective stress distributions within such slopes is necessary. Prediction of the dynamic performance of an embankment is intrinsically related to the static stress history of the embankment.

Figure 7 is a schematic section through a nonkeyed embankment composed of clayey fill subjected to varying groundwater levels. Shallow groundwater commonly accumulates in fills because of perching along less-permeable strata or older native soils buried beneath the embankment. In Figure 7 the relative change in effective stress (defined as the total stress minus the pore pressure) at two different positions in the

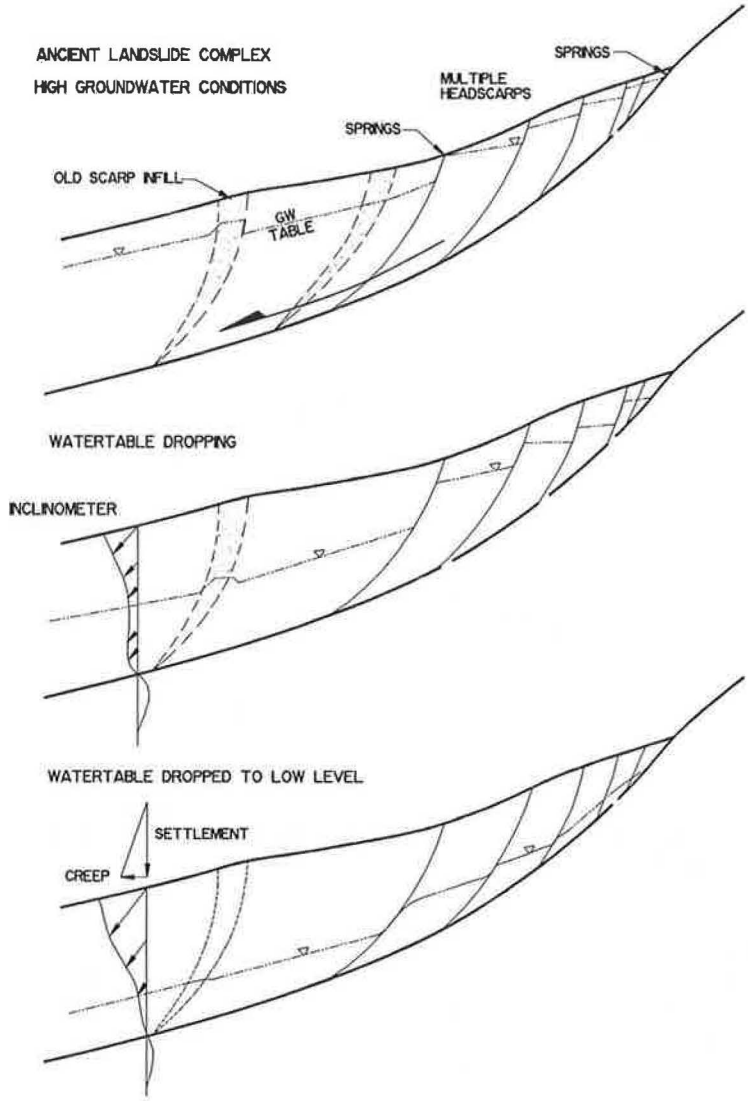

FIGURE 6 Consolidated shrink-induced settlement.

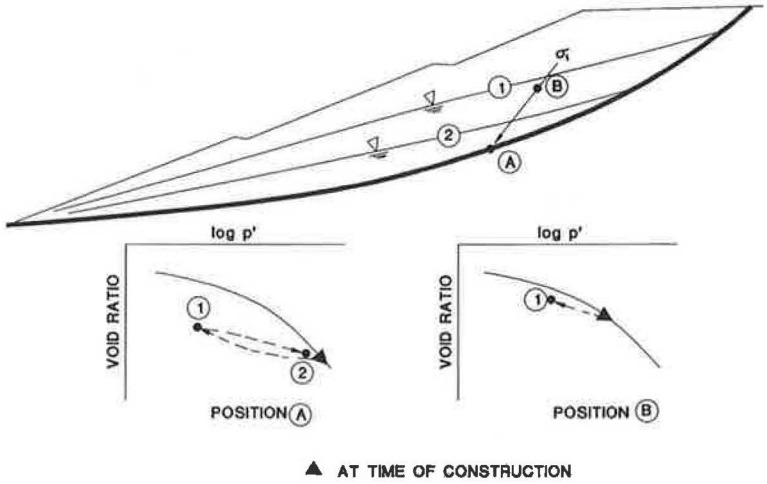

FIGURE 7 Effective stresses in valley-side fill during periods of high and low groundwater accumulation.

embankment is represented on conventional pressure-void ratio consolidation plots. Position *A* represents the approximate position of maximum effective stress at the time of construction. Note that this location lies on the virgin compression curve. If groundwater accumulates to a Level 1 (approximately five-eighths saturation of the embankment), the soil at Position *A*, through buoyance, swells along the rebound curve to Point 1, a reduction of two-thirds of the effective stress it originally felt. The amount of shear strength that could be mobilized at Position *A* is thereby greatly diminished through buoyance.

Soil situated somewhat higher in the embankment, at Position *B*, does not experience as dramatic a change, because of its less-confined position. At the time of compaction, it is just beginning to plot on the virgin consolidation curve, because of its overconsolidated nature (assuming a clay with moderate plasticity). When the water table rises to Level 1, the soil swells along its respective rebound curve. Through the process of cyclic swelling and shrinkage, the stress history of a clayey material (imprinted by mechanical compaction at the time of construction) is slowly erased (*12*). In addition, through such load cycling, intrinsic cohesion may be greatly diminished. Simple long-term saturation can also cause a loss of cohesion, as described by Morganstern and Eigerbrod (*13*) and demonstrated by Rogers and Pyles (*14*) (Figure 8).

As in the example of an ancient landslide, the prediction of settlement or lurching of embankments is strongly influenced by the in situ effective stress field. Figure 9 (*top*) shows the trajectories of maximum principal stress in valley-side sliver fill (unkeyed) embankments. A similar distribution of maximum-stress trajectories for keyed embankments is shown in the lower part of Figure 9. Note the influence of the slope face on the principal stresses, causing them to be inclined downslope.

Figure 10 shows expected settlement vectors in the same embankment as that in the upper half of Figure 9. Settlement vectors will tend to mimic the principal stress trajectories, engendering a significant horizontal component of movement much like a landslide. Outward surface manifestations of such movements—typically a down-dropped roadway shoulder, tension cracks aligned in an accurate manner around the deepest portions of the fill wedge, and asymmetric settlement of midslope drainage terraces—are very similar to incipient land slippage. Note how the horizontal component of motion can be fairly severe. Such slopes may continue to creep because of any number of environmental factors, such as a declining water table or groundwater feeding into discrete zones of the fill (through increased subsurface percolation generally caused by periods of excessive percolation).

If shrink-swell cycles continue unabated, embankment settlement due to creep will continue to manifest itself. Examination of inclinometer time-deformation histories suggests

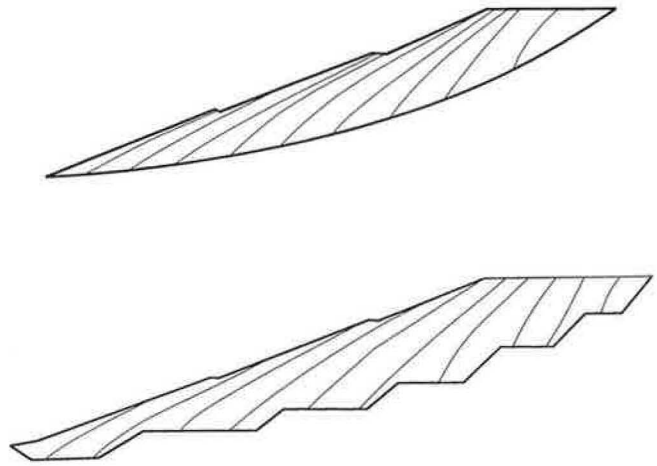

FIGURE 9 Principal effective stress distribution. *Top*: valley-side sliver fill. *Bottom*: keyed valley-side fill embankment.

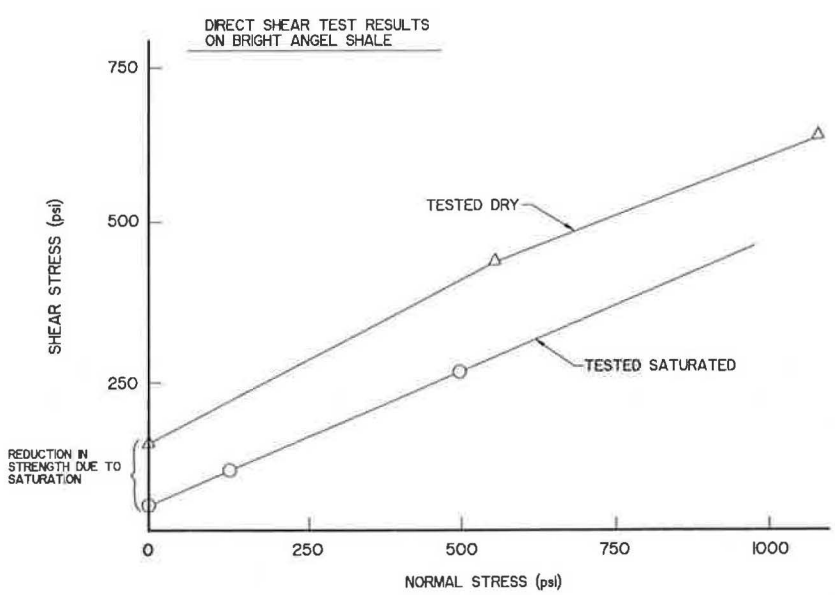

FIGURE 8 Reduction in effective cohesion of dense Cambrian-age shale subjected to shear testing parallel to bedding (*19*).

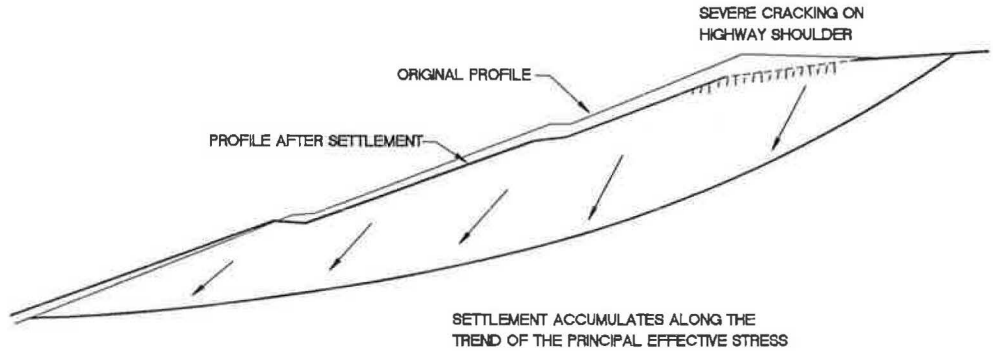

FIGURE 10 Expected settlement vectors in valley-side wedge fill.

that creep and settlement-induced strain is accumulated upslope, close to basal boundaries of the embankment, as shown in Figure 10. When the strain accumulates sufficiently to supersede the limiting strain of the soil, rupture will begin, generally along some of the shoulder tension cracks. In most embankments, this rupture is progressive from top to bottom of the slope. As accumulated strain levels approach 0.5 to 1.5 percent of the basal fill-contact length (L) as shown in Figure 11, rupture will progress at successively higher rates of strain until macrorupture occurs somewhere near the position indicated as MR. After shearing through the soil progresses to the vicinity of MR, sufficient driving forces are generally mobilized to rapidly overcome the remaining unsheared portion of the embankment toe. At this juncture, true landslide motion begins, where a semicoherent mass freely translates downslope. As macromotion of the entire slide mass continues, residual shear strength levels develop along the newly formed slide surface, thereby causing the resisting forces to degrade below prefailure peak strength levels. If the mobilized shear strength drops to residual values, landslide motion may continue for some period of time with less apparent connection to environmental conditions such as rainfall.

The progressive failure sequence described in Figure 11 has long been recognized by both highway and railroad engineers, who casually refer to such a sequence as a landslide that "creeped itself to failure" (*15*). Indeed, examples exist of large continuous slide motions without any apparent relation to weather extremes, for example, the Palolo Valley slide, first described by Peck (*16*), which has continued to move even through a significant drought from 1973 to 1988. In other instances, such as the celebrated slides along the Culebra Cut in the Panama Canal, an equal number of major bedrock slides reach their rupture points in dry years as in wet years (*17*).

With the above observations in mind, it would appear that virtually every slope or embankment possesses unique geologic, hydrologic, and construction history characteristics. These characteristics and the load history due to plasticity, creep, and cycling of entrained pore pressures likely serve to imbue each slope with a unique threshold with respect to future movement. In more colloquial terms, some slopes have "creeped" enough (through any number or combination of the above-cited factors) to be close to failure at any particular time. The concept of the continuously degrading slope safety factor was first proposed by Terzaghi (*18*) and has been cited by many others in explaining why some extremely large slides survive large storms and then are seemingly triggered by smaller ones (*16,19*).

DISCRIMINATING BETWEEN SETTLEMENT AND LANDSLIDING

Much confusion has arisen from recent postearthquake reconnaissances in which scores of ground cracks have been observed within and adjacent to large embankments and known ancient landslide complexes. The author believes that much of what has previously been described as seismically induced "landslippage" is actually seismically induced "settlement," the surface manifestations of which are strikingly similar but the analytical procedures for which are dramatically different.

Appreciating the similarities and variance between settlement and landslide-induced embankment deformations would appear particularly relevant to highway engineering maintenance, modification, and mitigation assessments. Illustrative case histories of embankments subject to both settlement (Figure 12) and landslide (Figure 13) are given.

Figure 12 shows a set of circumstances typical of a modern-day keyed highway embankment. The highway fill was keyed into underlying bouldery colluvium (Step 1). With time and seasonal precipitation, shallow groundwater can build up within the embankment (Step 2). At high water levels, the embankment may begin to exhibit outward manifestations of apparent downslope movement. At this juncture, most highway departments simply overlay the subsiding area with asphalt, but prudent engineers may opt for installing inclinometers to monitor long-term movement. In Step 3, inclinometers have been installed and the readings taken indicate a falling water table with apparent settlement and downslope motion. However, the apparent horizontal offsets measured in the inclinometers should mimic the fall of the water level (as shown in Figure 6) and will tend to decrease toward the brow of the embankment slope. This would be the classic subsurface pattern associated with simple consolidation and settlement of the embankment.

Figure 13 shows incipient landslide motion in a nearly identical geologic setting. As in the previous study, a keyed

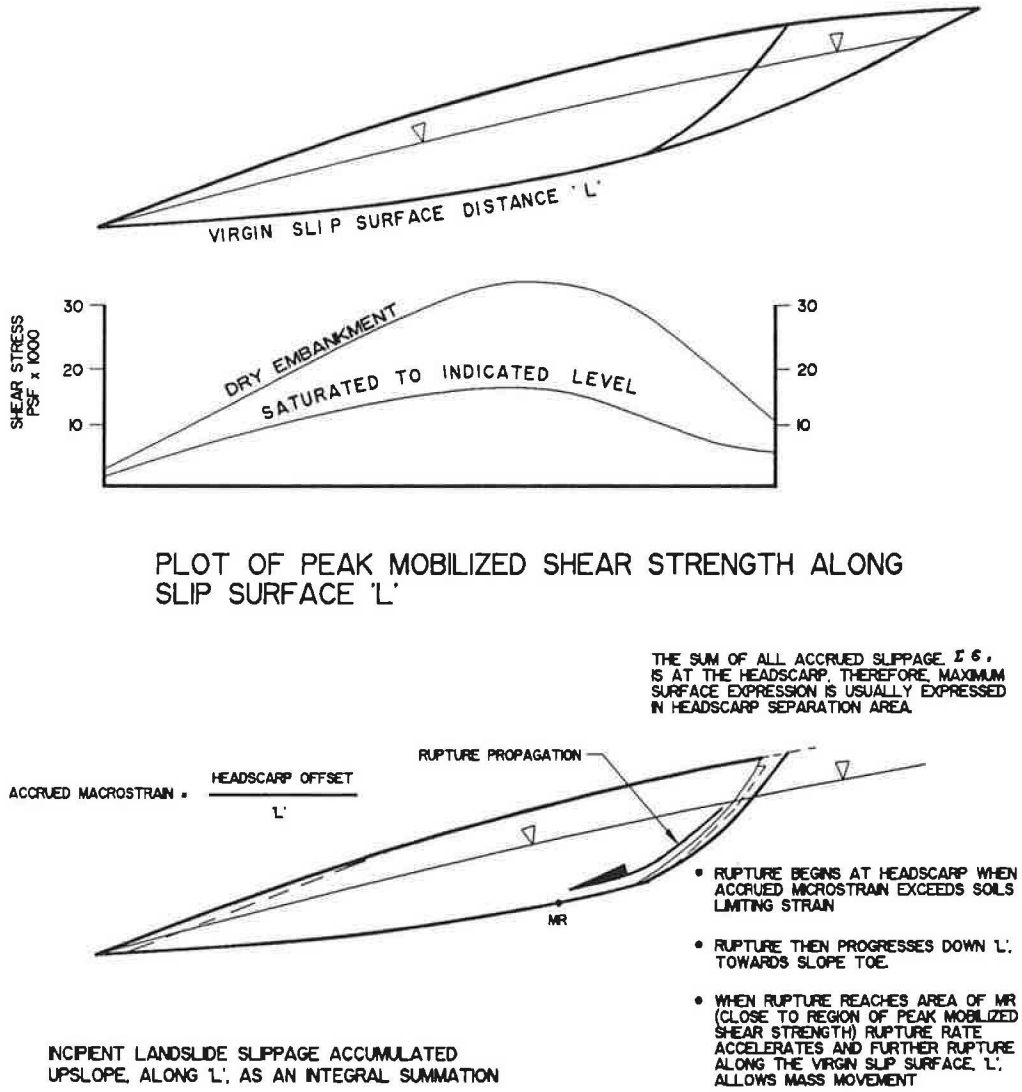

FIGURE 11 Plot of peak mobilized shear strength along slip surface *L*.

embankment was founded upon a bouldery colluvium with an overconsolidated clay matrix (Step 1). Once again, the roadway shoulder exhibited evidence of movement, so inclinometers were installed. In this instance, the ground surface settlement distribution was observed to be similar to that shown in Figure 12. The exception was that a sharp offset was noted in vicinity of the headscarp. Measured settlements to the roadway side of the headscarp were minimal when compared with those on the slope side. The headscarp separation appeared to mimic the approximate location of the embankment's upper key backslope. Inclinometer readings indicated a consistent increase in measured motion toward the brow of the embankment slope. This would be the classic subsurface deformational pattern associated with landsliding.

After macrotranslation of a landslide begins and the mass starts to translate downslope as a semicoherent mass, inclinometer casings are usually sheared off and little controversy remains as to what is occurring. Less often, large, translatory slide masses can creep along with similar movement levels manifest throughout the mass (although inclinometer readings still tend to be smaller at the toe and larger toward the headscarp). If an ancient translational slide is reactivated, less obvious precursory motion in terms of strain levels ascending upslope is measured.

CONCLUSIONS

Although engineering geologists are capable of identifying ancient landslide areas and highway embankments on a broad scale, the assessment of seismic instability potential is, at the present time, exceedingly difficult to predict. Permanent slope deformation under earthquake loading appears to be dependent on a large number of site-specific variables relating to geology, hydrology, seismology, and construction practices. However, certain slopes, through their geologic and topo-

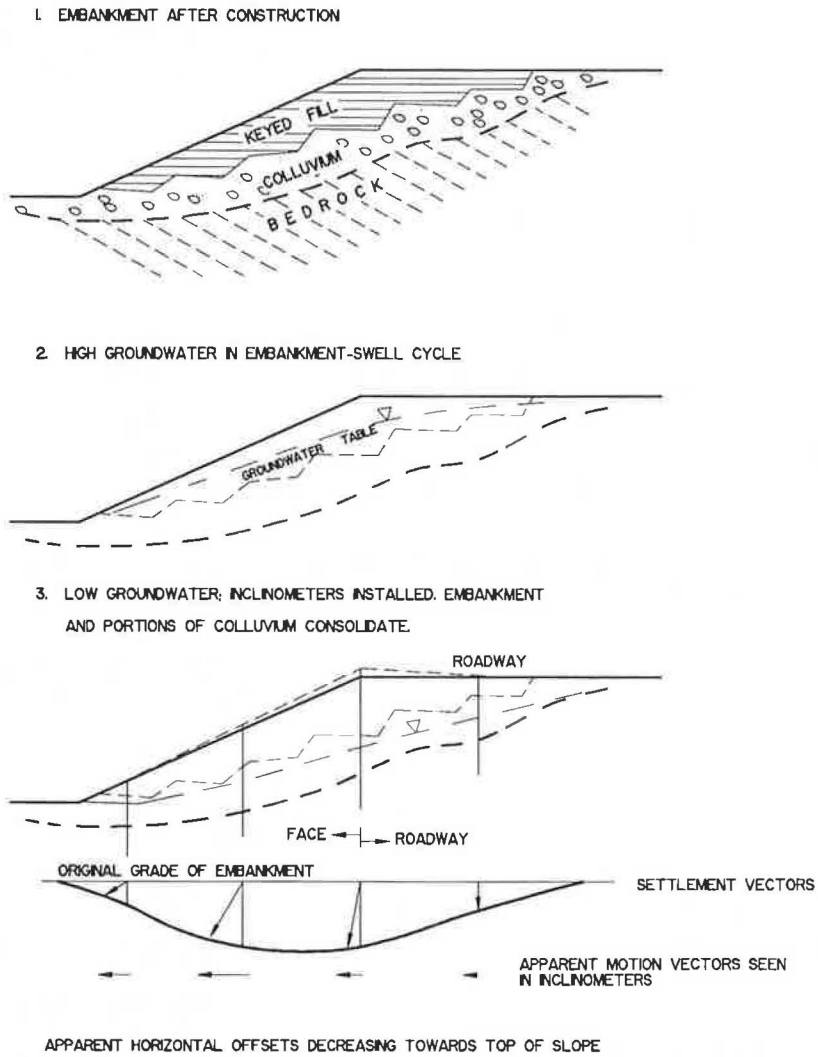

FIGURE 12 Slope strain by settlement.

graphic setting and previous site history, are likely more prone to failure because their static factors of safety have degraded to a point of marginal stability.

No simple limit equilibrium or pseudostatic loading relationship presently exists that can coprocess all of the geologic and seismologic variables. At this time, judicious engineering, geologic mapping and analyses, in situ instrumentation, and surface alignment surveys are the most proven approaches for defining potential problem areas. Ancient landslide reactivation can be analyzed using gross empirical estimates such as those posed by Wilson and Keefer (*20*) of the U.S. Geological Survey. Analysis of gross permanent embankment deformations can still be effected using simplified procedures developed by Makdisi and Seed (*7*) for earth dams, but these procedures should not necessarily be applied to more foreign situations, such as sanitary landfills. It is likely that an expansion of analytical techniques can be expected in the coming decade if federal funding for research in this area continues through the Federal Highway Administration, state transportation departments, or the National Earthquake Hazard Reduction Program.

ACKNOWLEDGMENTS

The author is grateful to the many individuals with whom he has associated who are astute observers of seismically induced slope movements, including David K. Keefer, Edwin Harp, and Ray Wilson of the U.S. Geological Survey; William C. Cotton; the late H. Bolton Seed; J. Michael Duncan; Ralph B. Peck; and Richard E. Goodman. John Walkinshaw solicited and kindly reviewed the article, which is a synopsis of a considerable body of information that the author is currently researching.

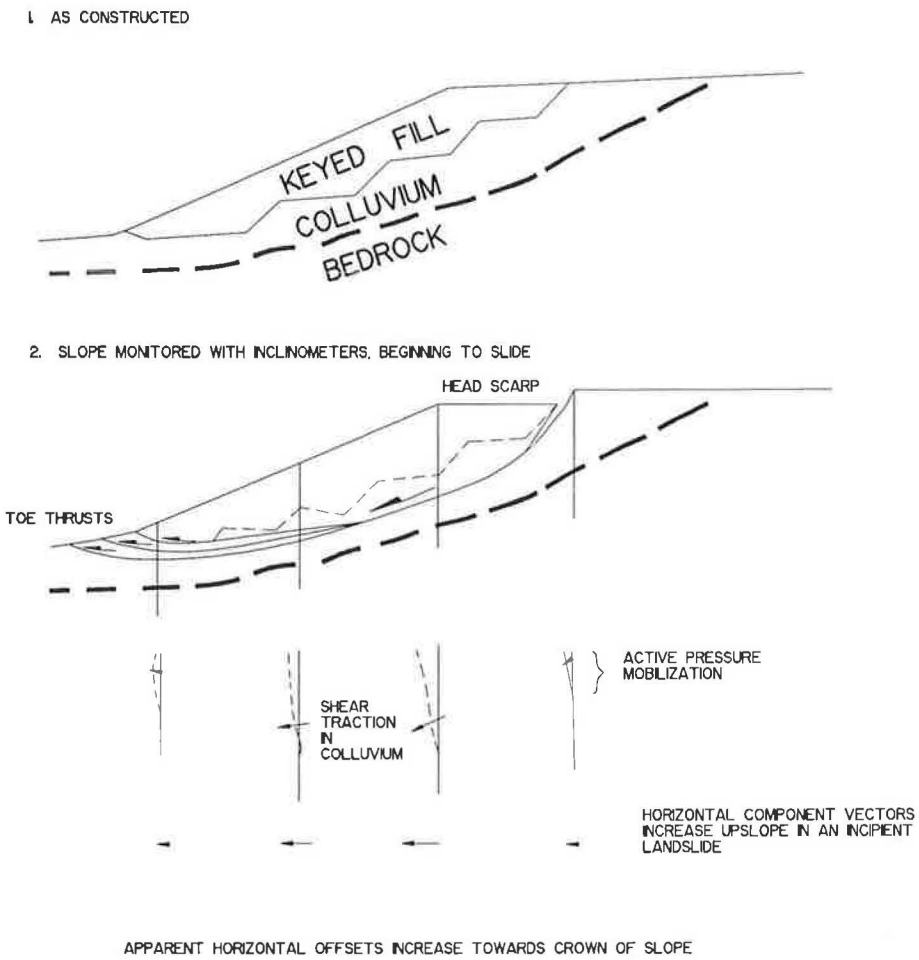

FIGURE 13 Embankment subject to landsliding.

REFERENCES

1. H. O. Wood. Distribution of Apparent Intensity in San Francisco. In *California Earthquake of April 18, 1906*, Carnegie Institution of Washington, D.C., Publication 87, Vol. 1, Part 1, 1908, pp. 220–254.
2. T. Naito. Earthquake-Proff Construction. *Bulletin of the Seismological Society of America*, Vol. 17, No. 2, 1927, pp. 57–94.
3. R. D. Borcherdt. Effects of Local Geology on Ground Motion Near San Francisco Bay. *Bulletin of the Seismological Society of America*, Vol. 60, 1970, pp. 29–61. R. D. Borcherdt and J. F. Gibbs. Effects of Local Geological Conditions in the San Francisco Bay Region on Ground Motions and the Intensities of the 1906 Earthquake. *Bulletin of the Seismological Society of America*, Vol. 66, 1976, pp. 467–500.
4. H. B. Seed and I. M. Idriss. Influence of Soil Conditions on Ground Motions During Earthquakes. *Journal of the Soil Mechanics and Foundation Engineering Division*, ASCE, Vol. 95, No. SM1, 1969, pp. 99–137.
5. H. B. Seed and I. M. Idriss. Influence of Soil Conditions on Building Damage Potential During Earthquakes. *Journal of the Structural Division*, ASCE, Vol. 97, No. SM9, Proc. Pap. 8371, Sept. 1971, pp. 1249–1273.
6. E. L. Krinitzsky and F. K. Chang. *State-of-the-Art for Assessing Earthquake Hazards in the United States: Report 25 Parameters for Specifying Intensity-Related Earthquake Ground Motions*. Misc. Paper S-73-1. U.S. Corps of Engineers, Waterways Experiment Station, Vicksburg, Miss., 1987, 83 pp.
7. F. I. Makdisi and H. B. Seed. *A Simplified Procedure for Estimating Earthquake-Induced Deformation in Dams and Embankments*. Earthquake Engineering Research Center Report UCB/EERC-77-19. University of California, Berkeley, 1977.
8. H. B. Seed and G. R. Martin. The Seismic Coefficient in Earth Dam Design. *Journal of the Soil Mechanics and Foundation Engineering Division*, ASCE, Vol. 92, No. SM3, May 1966.
9. R. E. Goodman. *Introduction to Rock Mechanics*. John Wiley & Sons, New York, 1980, 478 pp.
10. H. B. Seed. Earthquake Resistant Design of Earth Dams. In *Seismic Design of Embankments and Caverns: Proceedings of a Symposium*, Geotechnical Engineering Division, ASCE National Meeting, Philadelphia, 1983, pp. 41–64.
11. M. E. Hynes-Griffin and A. G. Franklin. *Rationalizing the Seismic Coefficient Method*. Misc. Paper GL-84-13. U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, Miss., 1984, 37 pp.
12. T. D. Stark and J. M. Duncan. Mechanisms of Strength Loss in Stiff Clays. *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 177, No. 1, Jan. 1991, pp. 139–154.
13. N. R. Morganstern and K. Eigenbrod. Classification of Argillaceous Soils and Rocks. *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 100, No. GT10, 1974, pp. 1137–1159.
14. J. D. Rogers and M. R. Pyles. Evidence of Cataclysmic Erosional Events in the Grand Canyon of the Colorado River, Arizona. In

Proceedings of the Second Conference on Research in the National Parks, San Francisco, National Park Service, Washington, D.C., Physical Sciences, Vol. 5, 1979, pp. 392–454.
15. G. E. Ladd. Landslides, Subsidence and Rockfalls. In *Proc., 36th Annual Convention*, American Railway Engineers Association, Chicago, Vol. 36, 1935, pp. 1091–1163.
16. R. B. Peck. Stability of Natural Slopes. *Journal of the Soil Mechanics and Foundation Engineering Division*, ASCE, Vol. 93, No. SM4, 1967, pp. 403–417.
17. D. F. McDonald. Panama Canal Slides. In *The Panama Canal Third Locks Project*: Department of Operations and Maintenance, Panama Canal Commission, Balboa Heights, Canal Zone, 1947, 73 pp.
18. K. Terzaghi. *Mechanisms of Landslides: Applications of Geology to Engineering Practice* (Berkey Volume), Geological Society of America, 1950, pp. 83–123.
19. J. M. Duncan. Prevention and Correction of Landslides. Presented at Sixth Annual Nevada Street and Highway Conference, Section II, April 7, 1971.
20. R. C. Wilson and D. K. Keefer. Prediction Areal Limits of Earthquake-Induced Landsliding. In *Evaluating Earthquake Hazards in the Los Angeles Region—An Earth Science Perspective*, U.S. Geological Survey Professional Paper 1360, 1985, pp. 317–346.

Publication of this paper sponsored by Committee on Soils and Rock Instrumentation.